Table D.9 — Preliminary stability criterion for different categories of berm breakwaters for modest angle of attack, $\beta = \pm 20°$

Category	H_o	H_oT_o
Non-reshaping	< 1,75 to 2,0	< 30 to 55
Reshaping, static stable	1,75 to 2,7	55 to 70
Reshaping, dynamic stable	> 2,7	> 70
NOTE The criterion depends to some extent on stone gradation.		

As mentioned before, it is recommended to design for reshaping static stable conditions.

D.2.2.2 Stability and reshaping of the berm breakwater head

Comparing results of tests by Van der Meer and Veldman[246], Burcharth and Frigaard[38],[39], Tørum[228] and Tørum et al.[233], it is concluded that if a berm breakwater is designed as a reshaped static stable berm breakwater, where $H_oT_o < 70$, it seems that by using the same profile for the head as for the trunk the head will be stable, with no excessive movements of the stones in the area behind the breakwater.

D.2.2.3 Rear side stability

Andersen et al.[8] arrived at the following relation for the necessary stone size D_{n50} on the rear side:

$$\frac{R_c}{H_{mo}}\sqrt{s_{02}} > \tan\alpha_f - \left(\frac{H_{mo}}{\Delta D_{n50}}\frac{1}{\sqrt{s_{02}}}\right)^{-1} \times \left(\frac{\mu\cos\alpha_r - \sin\alpha_r}{C_D + \mu C_L}\right) \quad \text{(D.32)}$$

where

R_c is the breakwater crest height;

H_{mo} $\approx H_s$;

C_D is the drag coefficient;

C_L is the lift coefficient;

α_f is the effective slope on the front side (Figure D.6);

α_r is the slope of rear side (Figure D.6);

μ is the friction factor = 0,9 for the material used by Andersen[8];

s_{02} $= 2\pi H_s/(gT_{02}^2)$.

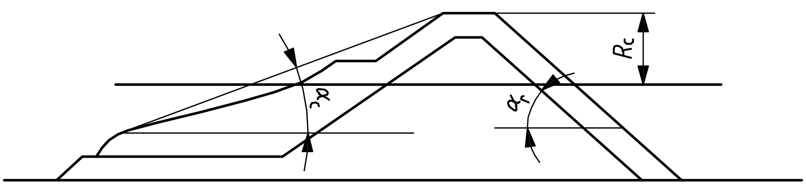


Figure D.6 — Definition of geometrical parameters for rear side stability

D.2.3 Wave overtopping

There are very few measurements of wave overtopping on berm breakwaters. Lissev[139] and Kuhnen[132] have measured average overtopping on a reshaped berm breakwater.

The Kuhnen data compare well with Lissev data, while the Van der Meer[245] relation for a conventional rubble mound breakwater show larger overtopping than the berm breakwater data. The results of Sigurdarson and Viggoson[198] indicate, as expected, that the time mean overtopping discharges for non-reshaped berm breakwaters are less than for reshaped breakwaters.

Burcharth and Andersen[45] gave preliminary results from the first systematic investigation on the wave overtopping of berm breakwaters.

D.2.4 Abrasion and stone crushing

When a berm breakwater reshapes, the stones suffer impact as they roll on the berm. ThIs impact may eventually lead to abrasion and/or breaking of the stones.

Abrasion seems not to be a problem for berm breakwaters as reported by Archetti et al.[13] for Icelandic berm breakwaters.

Impact breaking may be a problem for stones rolling on a berm breakwater. Tørum and Krogh[231] and Tørum et al.[232] developed a method of evaluating the suitability of stones from a specified quarry from the stone breaking point of view, when the stones roll on a reshaping berm breakwater. The method applies to reshaping static stable berm breakwaters. For this condition a stone will basically move once down the breakwater slope and come to rest at a lower level. The speed of the stone will vary as it moves along the slope, but it is anticipated that it will be subjected to one major impact if it hits another stone at maximum velocity. It is well known that a stone that does not break on the first major impact may break after many major repeated impacts, which could be the case for stones on a reshaped dynamically stable berm breakwater.

The probability of breaking of the stones can be evaluated by considering only two variables:

a) the impact energy;

b) the breaking energy required to break the stone.

Although both are dependent on the mass of the stone, velocity and strength are considered to be independent.

D.2.5 Local scour and scour protection

Assessment of local scour should preferably be based on experience. If lacking, then validated semi-empirical formulae or sediment transport theory can be used. Useful guidelines on scour and scour protection may be found in US Army Corps of Engineers Coastal Engineering Manual[235], Whitehouse[268], Sumer and Fredsøe[213], OCDI[163].

Annex E
(informative)

Wave actions on vertical and composite breakwaters

E.1 General

There are several types of wave action on vertical and composite breakwaters as discussed in 6.2.2. Among these types of action, those referred to in the main text are described in Clauses E.2 to E.5.

E.2 Extended Goda formula for wave action on main body of breakwater

Wave pressure exerted upon a front wall of the vertical or composite breakwater is assumed to have a linear distribution as shown in Figure E.1.

The elevation to which the wave pressure is exerted, denoted by η^*, is given by

$$\eta^* = 0{,}75(1 + \cos\beta)\lambda_1 H_D \tag{E.1}$$

where

β is the angle between the direction of wave approach and a line normal to the upright wall;

H_D is the wave height to be used in calculation as specified later.

The wave direction should be rotated by up to 15° toward the line normal to the upright wall from the principal wave direction in consideration of inaccuracy in defining the wave direction.

The pressure intensity is given by

$$p_1 = 0{,}5(1 + \cos\beta)(\alpha_1\lambda_1 + \alpha_2\lambda_2\cos^2\beta)\rho_w g H_D \tag{E.2}$$

$$p_3 = \alpha_3 p_1 \tag{E.3}$$

$$p_4 = \begin{cases} p_1\left(1 - \dfrac{h_c}{\eta^*}\right) & : \quad \eta^* > h_c \\ 0 & : \quad \eta^* \leqslant h_c \end{cases} \tag{E.4}$$

where

α_1, α_2, and α_3 are the coefficients given by Equations (E.5) to (E.7);

λ_1 and λ_2 are the pressure modification factors;

ρ_w is the density of seawater;

g is acceleration due to gravity;

h_c is the crest height of front wall above the still water level.

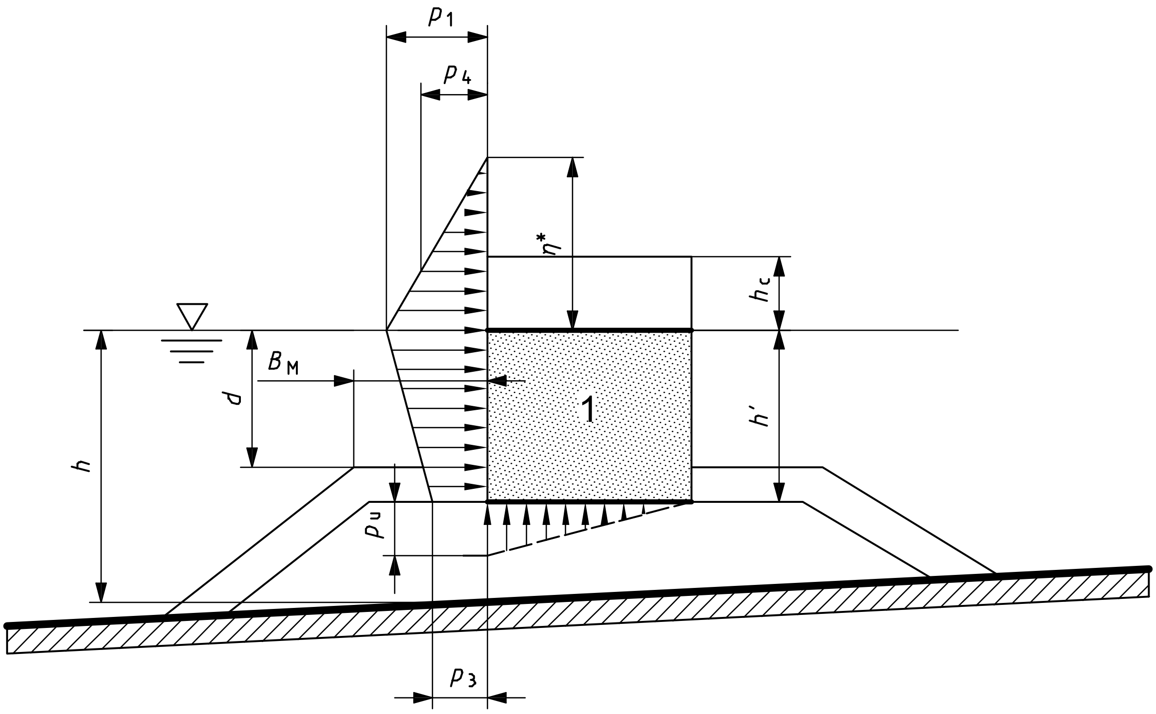
$$\alpha_1 = 0{,}6 + \frac{1}{2}\left[\frac{4\pi h/L}{\sinh(4\pi h/L)}\right]^2 \qquad \text{(E.5)}$$

$$\alpha_2 = \min\left\{\left(\frac{h_b - d}{3h_b}\right)\left(\frac{H_D}{d}\right)^2, \frac{2d}{H_D}\right\} \qquad \text{(E.6)}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{\cos h(2\pi h/L)}\right] \qquad \text{(E.7)}$$

where

- $\min\{a, b\}$ denotes the smaller one of a or b;
- d is the water depth on top of the armour units (or foot protection blocks);
- h is the water depth at the location of the front wall;
- h' is the water depth at the toe of the front wall;
- h_b is the water depth at an offshore distance of $\times$ 5 the significant wave height from the front wall;
- L is the wavelength at the water depth h.



Key

1 buoyancy

Figure E.1 — Distribution of wave pressure and uplift exerted on the main body of the breakwater

The uplift exerted on the bottom of the main body is assumed to have a linear distribution with the maximum intensity as follows:

$$p_u = 0,5(1+\cos\beta)\alpha_1\alpha_3\lambda_3\rho_w g H_D \tag{E.8}$$

The buoyancy is applied to the immersed part of the main body regardless of wave overtopping.

The pressure modification factors λ_1, λ_2, and λ_3 are given the values of 1,0 for standard vertical or composite breakwaters, but they are assigned smaller values for composite breakwaters covered with wave-dissipating concrete blocks (see OCDI[163], p. 109 and Tables VI-5-58 and VI-5-59 of Burcharth and Hughes[44]).

The wave height H_D is the height of the highest wave, which is taken as $\times$ 1,8 the design significant wave height $H_{1/3}$ when the breakwater is located outside the surf zone. Inside the surf zone, H_D should be evaluated by taking the random wave-breaking process into consideration. The wave period for evaluation of the wavelength L is the significant wave period of the design wave, which is approximately equal to $0,9T_p$ and $1,2T_m$ for wind waves.

For the cases in which consideration is needed for the exertion of impulsive breaking wave pressures, the coefficient α_2 must be replaced by $\alpha^* = \max\{\alpha_2; \alpha_I\}$, where α_I is the coefficient for impulsive breaking wave pressure and defined below.

$$\alpha_I = \alpha_{IH}\alpha_{IB} \tag{E.9}$$

where

$$\alpha_{IH} = \min\{H/d; 2,0\} \tag{E.10}$$

$$\alpha_{IB} = \begin{cases} \cos\delta_2/\cos\delta_1 & : \delta_2 \leqslant 0 \\ 1/(\cos h\delta_1 \cosh^{1/2}\delta_2) & : \delta_2 > 0 \end{cases} \tag{E.11}$$

$$\delta_1 = \begin{cases} 20\delta_{11} : \delta_{11} \leqslant 0 \\ 15\delta_{11} : \delta_{11} > 0 \end{cases} \tag{E.12}$$

$$\delta_{11} = 0,93\left(\frac{B_M}{L} - 0,12\right) + 0,36\left(0,4 - \frac{d}{h}\right) \tag{E.13}$$

$$\delta_2 = \begin{cases} 4,9\delta_{22} : \delta_{22} \leqslant 0 \\ 3,0\delta_{22} : \delta_{22} > 0 \end{cases} \tag{E.14}$$

$$\delta_{22} = -0,36\left(\frac{B_M}{L} - 0,12\right) + 0,93\left(0,4 - \frac{d}{h}\right) \tag{E.15}$$

where B_M is the width of the berm of the rubble mound in front of the main body.

The above formulae are due to Takahashi et al.[216].

Formulae for the total force and overturning moment can be found in Goda[88] p.139, and Table VI-5-55 of Burcharth and Hughes[44].

The Goda formula tends to overestimate the total wave loading on composite breakwaters by about 10 % with the coefficient of variation of 0,1 or so (see Takayama and Ikeda[217] and Table VI-5-55 of Burcharth and Hughes[44]). As such, the bias and uncertainty of the Goda formula should be taken in consideration in the probabilistic design of composite breakwaters.

E.3 Empirical formulae for minimum mass of armour units of rubble foundation

One of the available formulae for designing armour units of the rubble foundation of a composite breakwater is due to Tanimoto et al.[218] and is expressed by Equation (E.16). For other formulae, see Tables VI-5-45 to 48 of Burcharth and Hughes[44].

$$M = \frac{\rho_{\mathrm{s}}}{N_{\mathrm{s}}^3 (\rho_{\mathrm{s}} / \rho_{\mathrm{w}} - 1)^3} H_{1/3}^3 \qquad \text{(E.16)}$$

where

M is the minimum mass of an armour unit that is stable against the action of waves with a design significant height of $H_{1/3}$;

ρ_{s} and ρ_{w} are the densities of armour unit and seawater respectively;

N_{s} is the stability number to be calculated by use of Equation (E.17).

$$N_{\mathrm{s}} = \max\left\{1{,}8;\ \left(1{,}3\frac{1-\kappa}{\kappa^{1/3}}\frac{h'}{H_{1/3}} + 1{,}8\exp\left[-1{,}5\frac{(1-\kappa)^2}{\kappa^{1/3}}\frac{h'}{H_{1/3}}\right]\right)\right\} : B_{\mathrm{M}} / L' < 0{,}25 \qquad \text{(E.17)}$$

in which

$$\kappa = \frac{4\pi h' / L'}{\sinh(4\pi h' / L')}\kappa_2 \qquad \text{(E.18)}$$

$$\kappa_2 = \max\left\{0{,}45\sin^2\beta\cos^2\left(\frac{2\pi x}{L'}\cos\beta\right);\ \cos^2\beta\sin^2\left(\frac{2\pi x}{L'}\right)\right\} : 0 \leqslant x \leqslant B_{\mathrm{M}} \qquad \text{(E.19)}$$

where

$\max\{a;b\}$ is a function denoting the larger one of a or b;

h' is the water depth at which armour units are placed;

L' is the wavelength of the design significant wave period at the depth h';

β is the wave incident angle;

B_{M} is the berm width.

The factor 0,45 in Equation (E.19) is due to Kimura et al.[122] and accounts for the effect of the front slope of rubble mound.

E.4 Stability analysis of rubble mound and seabed against geotechnical failures

The rubble mound and the seabed foundation of a composite breakwater might experience geotechnical failure by the eccentric and inclined loading from the main body of the breakwater under wave action. The bearing capacity of the rubble mound and the seabed foundation can be analysed with circular arc calculations based on the simplified Bishop method (OCDI[163], pp. 277-278). The strength parameters of rubble stones of which the mound is composed are preferably estimated through large-scale triaxial compression tests, because they are affected by the stress level. For rubble stones generally used in port construction works however, their strength parameters can be represented with an apparent cohesion of c_{d} = 20 kN/m^3 and the internal friction angle of ϕ_{d} = 35°.

E.5 Local scour and scour protection

Assessment of local scour should preferably be based on experience. If lacking, then validated semi-empirical formulae or sediment transport theory can be used. Useful guidelines on scour and scour protection may be found in US Army Corps of Engineers Coastal Engineering Manual[235], Whitehouse[268], Sumer and Fredsøe[213], OCDI[166].

Annex F
(informative)

Wave action on coastal dykes and seawalls

F.1 Coastal dykes

F.1.1 Introduction

Coastal dykes are man-made sloped soil structures parallel to the shore to protect the hinterland against erosion and flooding. They comprise coastal dykes along the shoreline and estuary dykes in a river estuary. These dykes are characterized by flat slopes on the seaward side (usually 1:4 corresponding to an angle of 14° from horizontal and flatter) and on the shoreward side (usually 1:3 corresponding to an angle of 18,3° and flatter). Very often, berms are installed on the seaward and/or shoreward side of the dyke (e.g. dyke access roads). Coastal dykes are generally built of sand and/or clay and are covered by different materials such as grass, asphalt, stone revetments, etc. A summary of relevant hydraulic and geotechnical processes for coastal dykes is given in Figure F.1.

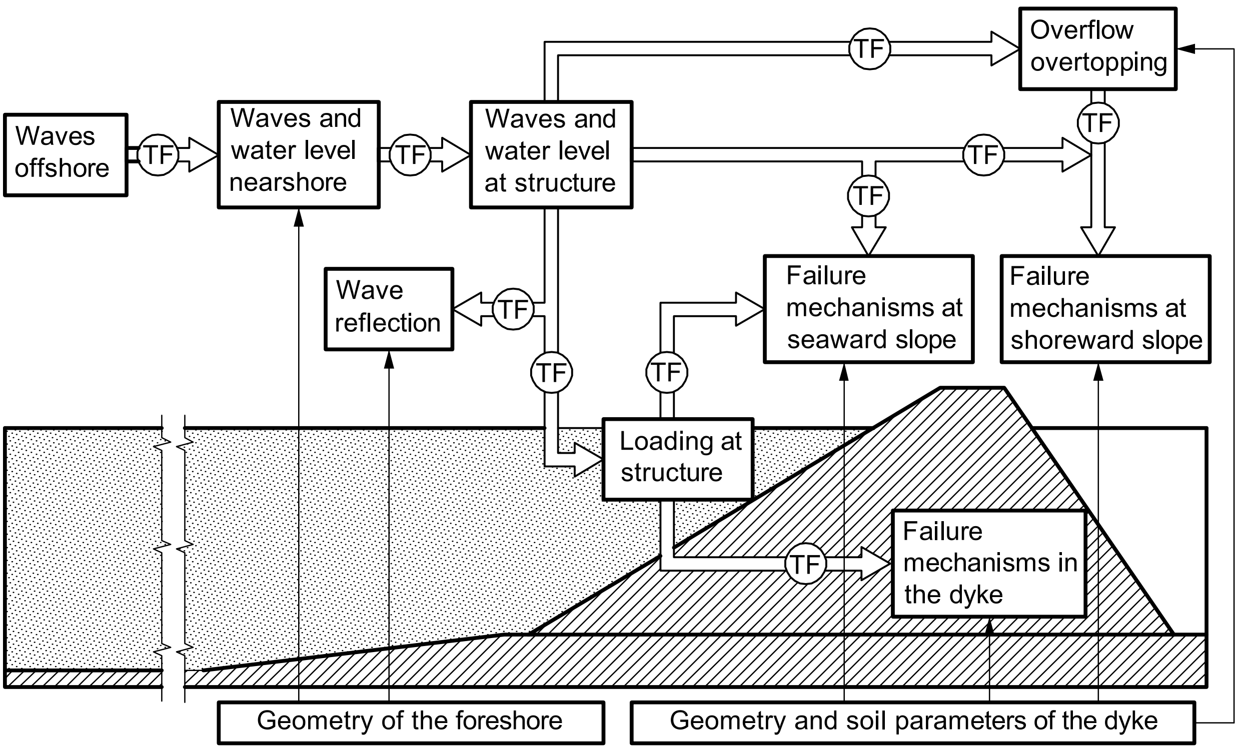


Figure F.1 — Overview of relevant hydraulic processes, loading processes and failure modes for coastal dykes

Figure F.1 shows that the hydraulic processes in the foreshore and the nearshore of the dyke are transferred to processes describing the loading at the structure and the overflow or overtopping, respectively. These "loading processes" can then be used to describe the failure mechanisms at the seaward side, the shoreward side and the interior of the dyke. Some guidance on "loading processes" and failure mechanisms is given in F.1.2 to F.1.5. The main design manuals available are EAK[71], OCDI[163], CIRIA/CUR[52], BSI[30].

F.1.2 Wave action on a seaward slope

For determination of wave run-up height the widely used definition of $R_{u,2\%}$ can be used which is defined as the height of the run-up tongue above still water level, which is exceeded by only 2 % of all waves. It can be determined using Van der Meer[245]:

$$\frac{R_{u,2\%}}{H_s} = 1{,}6 \times \gamma_b \times \gamma_f \times \gamma_\beta \times \xi_{op} \text{ with a maximum of } 3{,}2 \times \gamma_f \times \gamma_\beta \tag{F.1}$$

The surf similarity parameter or Iribarren number ξ_{op} (= tan $\alpha/s_0^{0,5}$) should be determined using the deep water wave steepness s_0 related to T_p and the significant wave height H_s. Empirical parameters γ_b and γ_β, describing the influence of a berm and the angle of wave attack are described in Van der Meer[245] and can be determined as follows:

$$\gamma_b = \begin{cases} 1-0{,}5\times\dfrac{B_A}{L_{berm}}\times\left(\dfrac{R_{u,2\%}+h_h}{R_{u,2\%}-H_s}\right) & \text{for } \dfrac{h_h}{H_s} < -1 \\ 1-\dfrac{B_A}{L_{berm}}\times\left(1-0{,}5\times\left(\dfrac{h_h}{H_s}\right)^2\right) & \text{for } \left|\dfrac{h_h}{H_s}\right| \leqslant 1 \\ 1-0{,}5\times\dfrac{B_A}{L_{berm}}\times\left(2-\dfrac{h_h}{H_s}\right) & \text{for } \dfrac{h_h}{H_s} > 1 \end{cases} \tag{F.2}$$

$$\gamma_\beta = 0{,}35+0{,}65\times\cos\beta \tag{F.3}$$

where

β is the angle of wave attack (0°, if perpendicular);

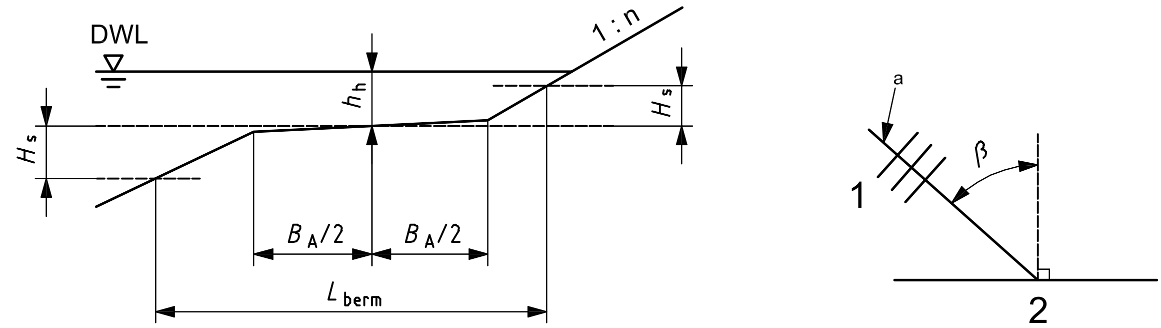
B_A is the horizontal width of the berm on the seaward slope of the dyke;

h_h is the height of water above the berm;

L_{berm} is the effective length of the berm as described in Figure F.2.

The empirical parameter γ_f can be taken from Table F.1. More parameters can be found in Van der Meer[245].

However, for shallow foreshores, breaking on the foreshore may occur, which changes the type of the wave spectrum. Under these conditions it might be advisable to use a different wave period ($T_{m-1,0}$) for calculation of the surf similarity parameter. Details can be found in Schüttrumpf and Van Gent[194].



Influence of berms **Influence of oblique wave attack**

Key

1 wave crests

2 shoreline

a Wave direction.

Figure F.2 — Reduction coefficients accounting for angle of wave attack and berm influence

Table F.1 — Overview of reduction coefficients γ_f accounting for roughness on coastal dykes

Type of dyke cover	Reduction coefficient γ_f
Asphalt	1,0
Grass	1,0
Basalt revetment	0,9

Wave run-down height is defined as the distance of water level elevation during wave trough to still water level. It can be determined using Schüttrumpf[191] or Van Gent[239]. The thickness of the water layer on the dyke changes with the elevation over the still water level and can be determined for the seaward side of the dyke using Schüttrumpf (for crest and shoreward side see F.1.4). Mean run-up velocities on the seaward side, the crown and the shoreward side of the dyke can be estimated using Schüttrumpf and Van Gent[194] or Schüttrumpf.

Infiltration in the dyke body may result from excessive overtopping (from the shoreward side) or wave run-up and overtopping on the seaward side and the crown. Investigation has been made to determine whether this is mostly dependent on the mean layer thickness of the water body on the dyke, see Kortenhaus[124].

The phreatic water level in the dyke body will influence the geotechnical parameters of the dyke in the long term and is therefore more relevant for long-lasting water levels in front of the dyke. The duration for seepage through the dyke body can be estimated using e.g. Kortenhaus (as above).

Wave-induced uplift forces beneath the revetment or cover layer are very relevant for removal failure of revetments and therefore need to be duly considered. They can be estimated using e.g. Bezuijen and Klein-Breteler[23].

Erosion of grass and clay material at the seaward side is difficult to predict and, to date, only empirical formulae exist. However, these formulae are only validated by a very limited number of model tests and variations of relevant parameters may yield different results. The most promising approach is to predict the duration needed for erosion of the respective layers. More detailed information regarding erosion of grass layers is given in TAW[221]; some details on erosion of clay can be found in Möller et al.[156]. Regional data may be available and should then be used because soil, vegetation types and agricultural and construction practices have a large influence on erosion rates.